Murchison Flood Mapping

Study Report

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PROJECT DETAILS

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Cover Photo: Upstream view of the Bendigo-Murchison Road bridge over the Goulburn River – Photo courtesy of the Goulburn Broken CMA

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EXECUTIVE SUMMARY

The Murchison Flood Mapping Study was commissioned by the Goulburn Broken CMA to update the rating curve for the Goulburn River at Murchison using a two-dimensional hydraulic model, produce flood mapping for Murchison and to produce flood intelligence to update the Municipal Flood Emergency Plan. The resolution of the flow estimation is particularly important to inform inputs into the current Shepparton Mooroopna Flood Mapping and Flood Intelligence Project and the Regional Goulburn River Flood Mapping Project.

Revision of Rating Curve

A two-dimensional hydraulic model was developed for the reach of the Goulburn River extending approximately 2.7 km south of Murchison and north east approximately 8 km. The model was run for several steady state flows to calibrate the model to the previous rating curve for in channel flow and to extrapolate the rating curve for larger flows. The model was run for 12 incremental gauge heights from 9.0 to 12.2 m, including the 1% AEP gauge height. The revised estimate for the 1% AEP gauge height is 11.9 m, which was derived by applying the updated rating curve from this study. A flood frequency analysis was undertaken using the revised rating curve, with the results summarised in Table E-1.

Table E-1 Results of Flood Frequency using Revised Rating Curve

AEP	ARI (1 in X years)	Adopted Peak Flow (ML/d)	Gauge Height (m)
20%	5	49,100	9.9
10%	10	69,000	10.4
5%	20	90,900	10.8
2%	50	123,900	11.4
1%	100	152,600	11.9
0.5%	200	166,500	12.1
0.2%	500	196,900	12.4

Flood Mapping

Flood mapping for Murchison was completed for flood events for the 12 incremental gauge heights. Depth maps with water level contours are included in Appendix A. These maps have been formatted using the Municipal Flood Emergency Plan guidelines so they can be included in the update of the plan.

Flood Intelligence

Levee Freeboard

The levee just downstream of the Bendigo-Murchison Road bridge provides protection from river flooding. This levee has a minimum crest elevation of 121.04 m AHD. The 1% AEP has a water level near the levee of 120.43 m AHD, which means the levee has a freeboard of 610 mm for the 1% AEP event. The 1916 event had a water level of 120.73 m AHD near the levee, giving a freeboard of 310 mm.

Surcharge through Town Drainage

There is a possibility that surcharge from the river may inundate low lying areas of the town during large flood events. Analysis showed that areas of the town could potentially be inundated for the five largest modelled flood scenarios.

Flood Mitigation

For the largest modelled event, the hydraulic model shows a flow path through the western side of Murchison. The source of this flow path is due to a breakout from the Goulburn River upstream of town through a shallow depression. Given the long lead times prior to a Goulburn River flood in Murchison, there is time to coordinate sandbagging of this breakout to prevent flooding through town.

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1. INTRODUCTION

Water Technology was commissioned by the Greater Shepparton City Council to undertake the Shepparton Mooroopna Flood Mapping and Flood Intelligence Project. This study involved detailed hydrological and hydraulic modelling of the Goulburn River, Seven Creeks and the Broken River, flood mapping, assessment and treatment of flood risk.

During the course of the study it was uncovered that the gauge at Murchison had recently undergone a revision of the rating curve which varied significantly from previous rating curves at Murchison. After discussions with the Department of Environment & Primary Industries (DEPI) and the Goulburn Broken CMA, this project was commissioned to develop a hydraulic model of the gauge on the Goulburn River at Murchison to better estimate high flows above recorded flow gauging.

2. DATA REVIEW

A large amount of information was made available for the Shepparton-Mooroopna Flood Mapping and Flood Intelligence Project. A selection of this data in the vicinity of Murchison has been used for the purposes of this study.

2.1 Previous Studies

The Shepparton Mooroopna Floodplain Management Study (SKM, 2002) was a major study that considered the issues of flooding in Shepparton and Mooroopna, part of which is relevant to Murchison, and as such was reviewed in detail.

2.2 Digital Elevation Models and Survey

2.2.1 LiDAR Data

Two sets of LiDAR data for the region were made available for this study, supplied by the Goulburn Broken CMA and DEPI:

- Fugro Spatial Systems (FSS) – 1 m and 5 m DEM
    - Flown in 2007
    - Vertical accuracy ± 0.1 m
- Index of Stream Condition (ISC) – 1 m DEM
    - Flown in 2011
    - Vertical accuracy ± 0.15 m

An analysis of these two datasets showed that the FSS LiDAR was slightly lower on average than the ISC LiDAR. An adjustment of 100 mm was made to the FSS 1 m and 5 m DEMs to increase the level to match the ISC LiDAR. The justification for this adjustment is discussed further in the Shepparton-Mooroopna Flood Mapping and Flood Intelligence Project Hydrology and Hydraulic Calibration Report (Water Technology 2014).

The ISC data follows the alignment of major waterways, but doesn't extend far onto the floodplain. This data set was found to be the most consistent with the feature survey, whereas the FSS data was approximately 100 mm lower than the feature survey. The FSS data extends further onto the floodplain and helps to fill in a few of the gaps that the ISC data has missed.

2.2.2 Feature Survey

A small amount of feature survey was made available for the levee in Murchison that is located between the Goulburn River and Stevenson St, just downstream of the Bendigo-Murchison Road bridge. This survey was provided by the Goulburn Broken CMA.

2.3 Structures

There are two bridges over the Goulburn River within the study area (Bendigo-Murchison Road and Railway Line). The Bendigo-Murchison Road includes five waterway openings through the causeway to the east of Murchison and the Goulburn River. These structures were surveyed by the Goulburn Broken CMA, with structure details and photographs supplied to Water Technology for this study shown in Appendix B. The railway bridge has a flood level mark on one of the eastern piers from 1916. Accurate survey of this flood mark was not available for this study, so it has been used as a guide only.

2.4 Imagery

A recent aerial photo from 14th December 2009 was used for mapping purposes as a background image. This image was supplied by the Goulburn Broken CMA.

Flood imagery of any significant historic events was not available for this reach of the Goulburn River.

2.5 Streamflow Data

The streamflow gauge on the Goulburn River at Murchison (405200) is located within the study area. A major objective of this study was to revise the rating curve at this gauge for high flows, so any historic high flows were treated with caution. There is a gap in the period of record for this gauge from 1967 to 1984. This period can be infilled with data from the gauge at the Goulburn Weir, which is approximately 16 km upstream of Murchison. The details of these two gauges are summarised in Table 2-1.

Table 2-1 Streamflow Gauge Details

Station Name	Station No.	Area (km^2)	Period of record
Goulburn River at Murchison	405200	10,772	June 1881 to March 1967 November 1984 to current
Goulburn River at Goulburn Weir	405253	10,627	March 1967 to October 1985

3. HYDRAULIC MODEL DEVELOPMENT AND CALIBRATION

3.1 Hydraulic modelling framework

3.1.1 Overview

A hydraulic model was required to simulate the flow behaviour across the Murchison floodplain balancing excessive model simulation times and topographic resolution. The final hydraulic modelling framework comprised a two-dimensional (2D) hydraulic model which represents broad scale channel floodplain features, coupled with one-dimensional (1D) representations of structures such as bridges and culverts.

A TUFLOW model of the study area was constructed. Inputs for the TUFLOW model include:

- Topography data;
- Stream gauge data;
- Site roughness;
- Boundary conditions; and
- Structure details.

3.1.2 Hydraulic Model Capabilities and Uncertainties

There are numerous contributing factors to the ultimate output uncertainty in a complex hydraulic modelling exercise such as that undertaken for this study. Some of the uncertainties relate to the data inputs, whilst others are dependent on the numerical modelling processes itself. Sources of output uncertainty related to the input data for the hydraulic modelling include:

- Topographic data
- Definition of hydraulic controls/structures
- Observed flows and water levels for model calibration

Sources of uncertainty related to the hydraulic modelling process include:

- Model schematisation and set-up (bridge and culvert representation, grid resolution)
- Model parameters such as computational time-steps, surface-friction and other energy-loss parameters (expansion/contraction coefficients and eddy viscosity for example)
- Model numerical and computational schemes – these relate to the ability of the model to replicate the physics of free-surface flow in channels and over land
- Floating point accuracy of computing resources (round-off error)

There is a wide variation in the magnitude of the impact associated with each source of uncertainty. In order to identify the most significant sources of uncertainty it is possible to consider items as either first or second order magnitude, where second order items are of a significantly smaller magnitude compared to first order items and can generally be ignored. The first order sources of error are survey data, definition of hydraulic controls/structures, flow gauging data, observed flood levels for calibration, model schematisation, and model parameters.

The model development process can only address uncertainties arising from the following aspects:

- Definition of hydraulic controls/structures
- Model schematisation and set-up (location and spacing of cross-sections, grid resolution)
- Model parameters such as computational time-steps, surface-friction and other energy-loss parameters

3.2 Model Development and Schematisation

This section defines the scope of the hydraulic analysis, details the hydraulic model construction, and discusses the hydraulic model calibration.

3.2.1 2D Hydraulic Model Schematisation

Grid Extent and Resolution

The 2D model extends south of Murchison approximately 2.7 km, north east approximately 8 km, and has a grid resolution of 5 m. The width of the modelled floodplain is on average approximately 2 km, resulting in just fewer than 700,000 active grid cells. The extent of the model is shown in Figure 3-1.

Topography

The model topography was derived mainly from the 1 m ISC LiDAR DEM. There were two small sections of floodplain that weren't covered by the ISC data, being the meander cutoff just south of the Murchison Township and a small section of floodplain on the south side of the Goulburn River just upstream of the model downstream boundary. These two sections were filled in using the adjusted FSS LiDAR DEM. The TUFLOW hydraulic model resampled both 1 m LiDAR datasets to 5 m resolution model topography, prioritising the ISC data over the FSS data. The resultant grid is shown in Figure 3-1. Note that the model was extended downstream to ensure the downstream boundary did not have an undue influence on modelled water levels in town.

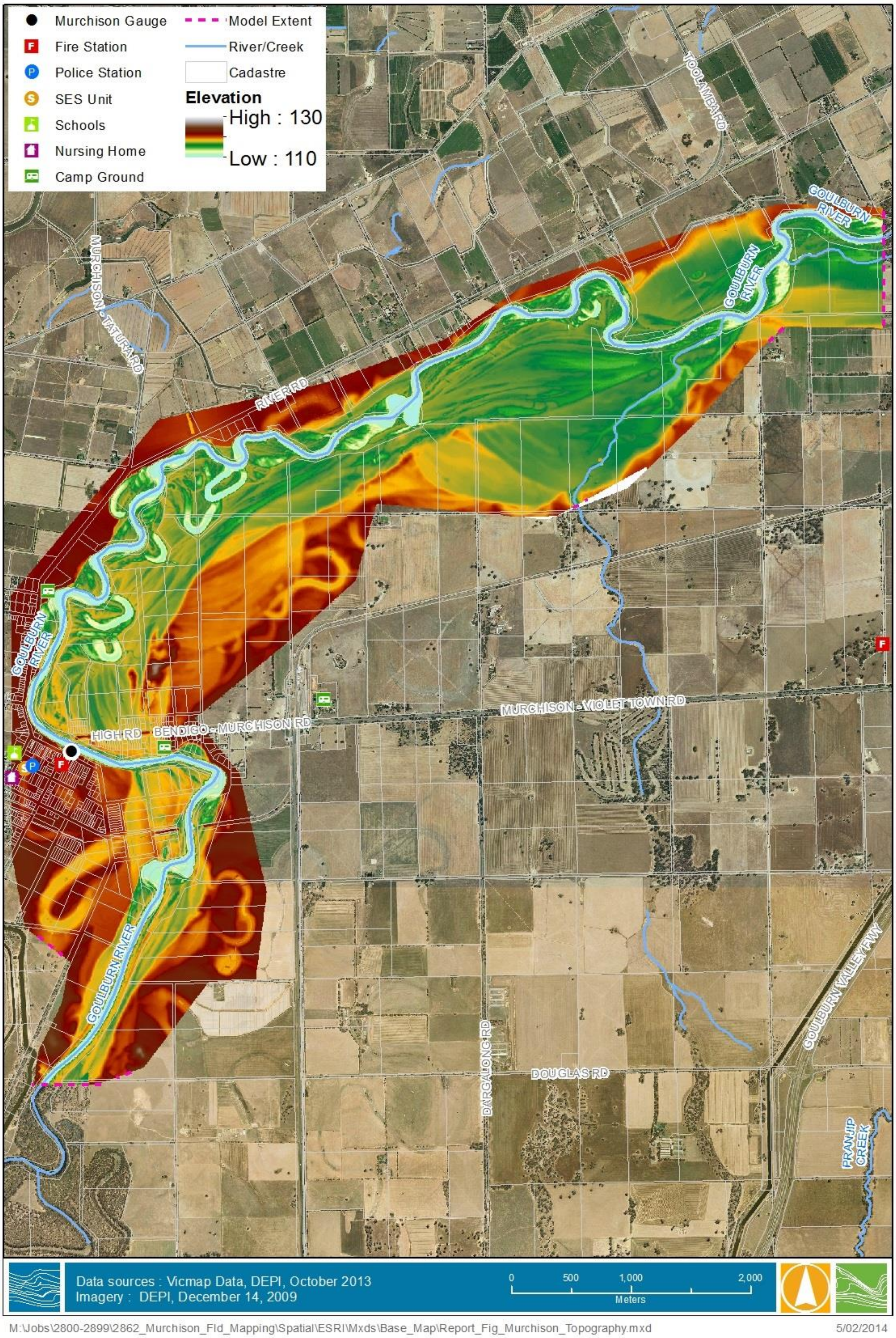

Figure 3-1 Model Topography

Boundary Conditions

The boundary conditions for the 2D hydraulic model were simply an upstream steady state inflow and a downstream tailwater level. The steady state inflows were estimated based on the desired water level to be modelled at the gauge. The downstream tailwater condition was estimated for each steady state flow based on the hydraulic gradient of the Goulburn River. From an interrogation of the water level in the ISC LiDAR DEM, it was found that the water level at the downstream boundary was approximately 1.68 m below the level at the gauge location. Therefore, for each desired gauge level to be modelled, the tailwater condition was assumed to be 1.68 m below this level. A sensitivity analysis for this assumption was undertaken and has been described in Section 3.3.5.

Bridges

There are two bridges that cross the Goulburn River, being the Bendigo-Murchison Road bridge in town and the railway bridge upstream. There are also another three bridges and two culverts under the Bendigo-Murchison Road causeway on the eastern floodplain. These five bridges have all been modelled within the 2D schematisation, whilst the two culverts have been modelled in 1D.

The TUFLOW hydraulic model utilises blockage and form losses on the 2D cell faces to simulate the flow through bridge structures. This is the recommended schematisation for structures that are larger than the 2D cell size. A form loss coefficient of 0.2 was assumed for all of the bridges, and the blockage percentage represents the bridge piers, which was calculated from the structure drawings provided by the Goulburn Broken CMA. Table 3-1 summarises the structure details and Appendix B contains the drawings supplied by the Goulburn Broken CMA, showing the location of the structures along with photos and indicative cross sections.

Table 3-1 Bridge Structure Model Details

Stucture ID	From Loss Coefficient	Blockage
Road Bridge	0.2	20% (bridge abutments only)
Railway Bridge	0.2	10%
SN4881	0.2	6%
SN4883	0.2	6%
SN4884	0.2	6%

Roughness

The variation in hydraulic roughness within the study area was schematised as two separate roughness layers, one representing all of the roads and the other representing the various hydraulic roughness values (e.g. floodplain, channels, vegetation etc). Areas with different roughness types were identified using aerial photographs and Vicmap data layers. The values adopted for the two-dimensional hydraulic model are summarised in Table 3-2 below. These values were based on standard industry roughness values and were modified during the calibration process. The values adopted are reasonable estimates of hydraulic roughness given the floodplain condition.

Table 3-2 Hydraulic Roughness Parameters

Land Type	Roughness (Mannings 'n')
Roads	0.025
Main Roads	0.02
Crops	0.05
Medium Density Vegetation	0.07
High Density Vegetation	0.08
Residential	0.06
Industrial	0.06
Cleared Land/Open Space	0.04
Goulburn River Channel	0.045
Pipes/Culverts	0.013

3.2.2 1D Hydraulic Model Schematisation

Culverts

There are only two significant culverts within the study area that have both been modelled in 1D. The culverts are located either side of the bridge with ID SN4881 along the Murchison causeway. Table 3-3 summarises the structure details and Appendix B contains the drawings supplied by the Goulburn Broken CMA.

Table 3-3 Culvert Structure Details

Stucture ID	Type	Dimensions
SN4880	Box Culvert	3 x 2.4 m (w) x 1.8 m (h)
SN4882	Pipe	8 x 1.8 m

Roughness

The hydraulic roughness for the two culverts assumed a Mannings 'n' of 0.013, as described in Table 3-2.

3.3 Hydraulic Model Calibration and Validation

3.3.1 Calibration Approach

Water level and flow gauging data was available for model calibration at the Murchison gauge, along with other observed water levels from the 1974 and 1916 flood events. The gauge has been in operation since 1887 and the rating curve has been developed with numerous gaugings over its history (Figure 3-4). The historic flood levels for 1974 were obtained from the Victorian Flood Database, with the reliability of this data considered by the providers to be either medium or low. Inspection of the flood levels found a wide variation in recorded levels, therefore no attempt to calibrate to this data was made. A comparison of the observed and modelled water levels is discussed in Section 3.3.6. There was only one recorded level from the 1916 flood event provided, which was an unsurveyed mark on the railway bridge upstream of Murchison that was unsuitable for calibration.

The model was calibrated by running a series of steady state flows within the high-confidence section of the rating curve, and comparing the modelled levels to the rating curve.

3.3.2 Current and Historic Rating Curves

The current rating curve (version 73.00, valid March 2012 to present) and three historic rating curves valid in 1974, 1993 and 2010 are shown in Figure 3-2 and Figure 3-3. The rating curves for 1974, 1993 and the present are all very similar, and are identical above the 50,000 ML/d discharge. The 2010 curve is quite different. It is not known why such a different rating curve was applied to the 2010 event, although it may have been the result of event-based monitoring by Thiess.

The current rating curve is considered reliable up to 184,000 ML/d and has been extrapolated above that point.

The historic gaugings used to construct the rating curve are shown in Figure 3-4. Note that the highest gauging is at around 100,000 ML/d. It appears there has been some extrapolation in the "reliable" section of the rating curve above the highest gauged flow.

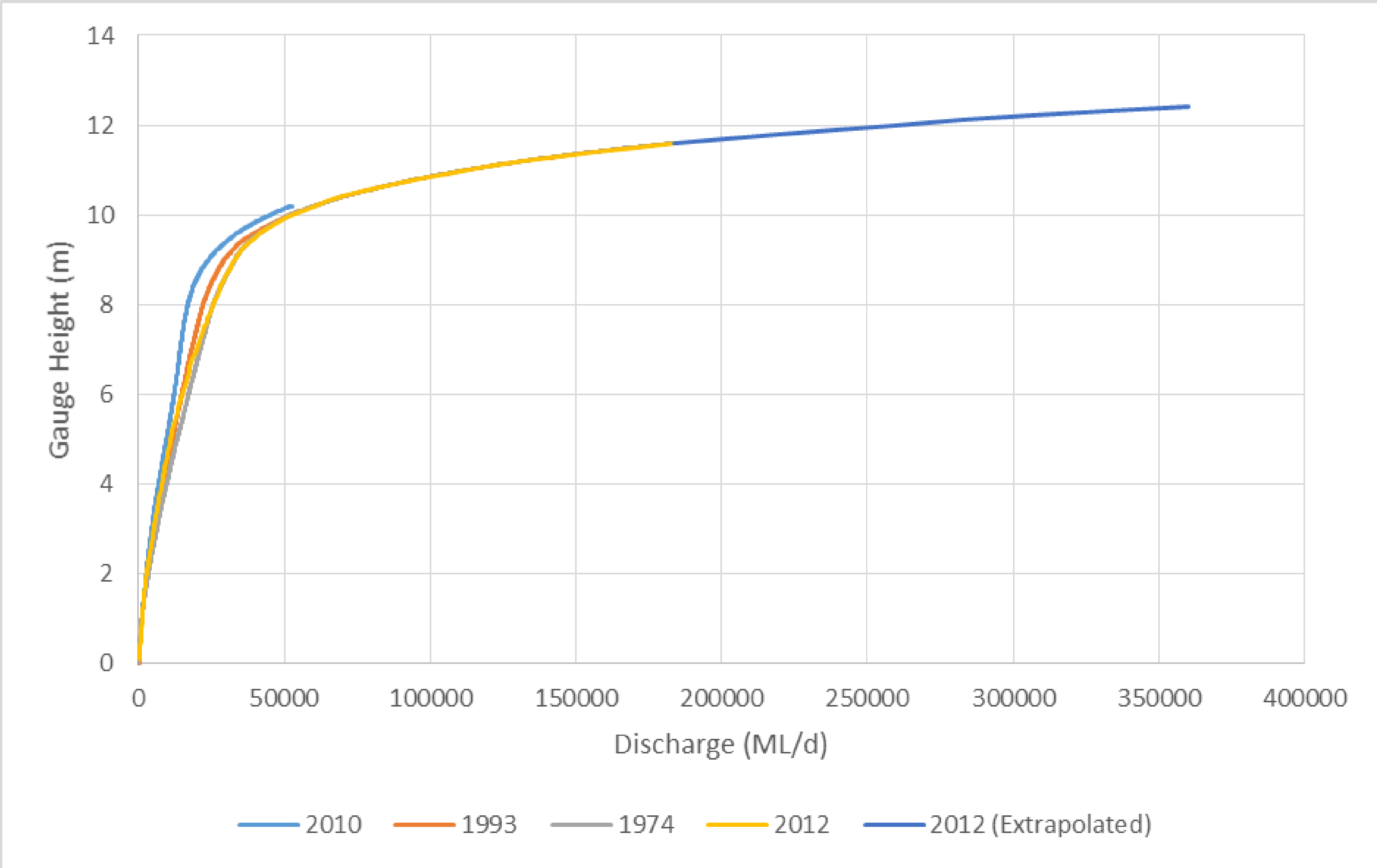

Figure 3-2 Current and historic rating curves

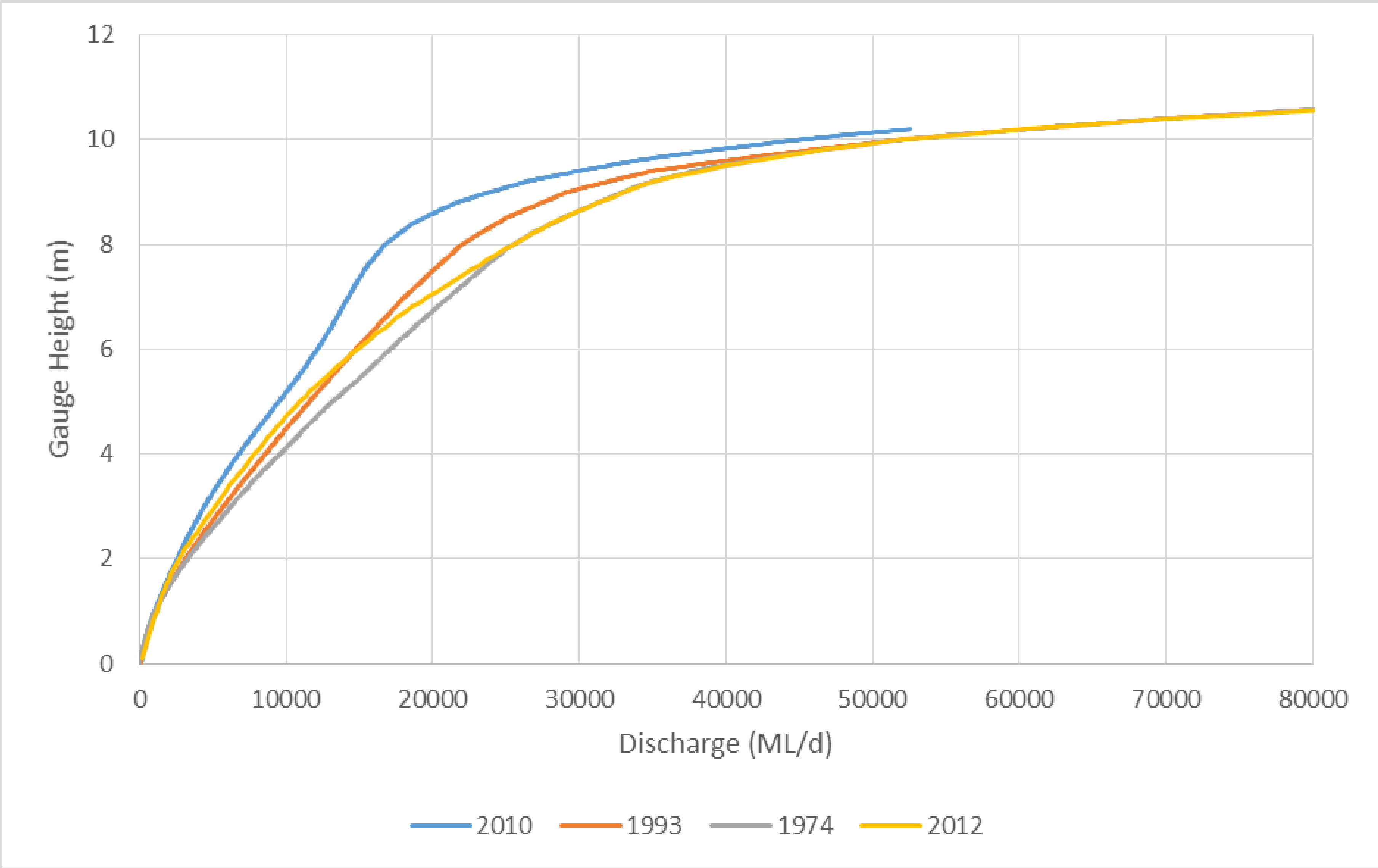

Figure 3-3 Current and historic rating curves (low flow section)

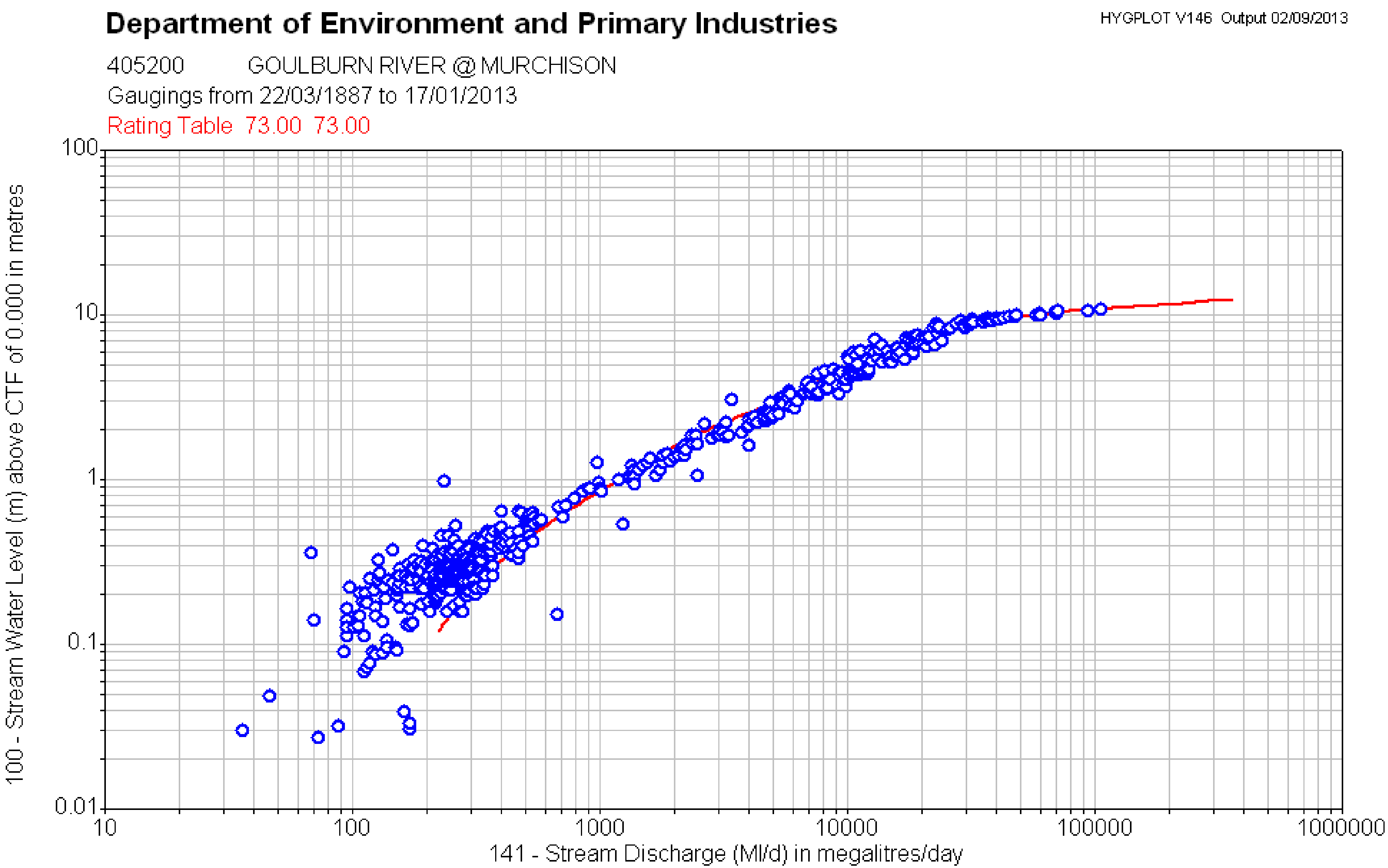

Figure 3-4 **Gaugings and current rating curve**

3.3.3 Calibration Flows and Levels

The recorded water level and peak flow for three historic events are included in Table 3-4. For calibration, a series of steady state flows were run in the hydraulic model, including these event peak flows, up to the upper limit of the non-extrapolated rating curve (184,000 ML/d). The flows that were included are given in Table 3-5, along with their corresponding water levels.

The tailwater level at the model outlet was set at 1.68 m below the expected level at the Murchison gauge, based on interrogation of water surface slope in the ISC LiDAR DEM.

Table 3-4 Historic event flows and recorded levels

	Peak Gauge Height (m)*	Peak WL (m AHD)	Peak flow (ML/d)	Rating curve
1974	11.29	119.97	142,000	1974**
1993	10.27	118.95	63,500	1993**
2010	10.14	118.82	50,200	2010

** Gauge zero 108.679 m AHD*

*** 1974, 1993 and current (73.00) rating curves are identical above 52,000 ML/d or 10 m gauge height*

Table 3-5 Calibration flow ramp

Flow	Level (current rating curve)		Tailwater level
(ML/d)	(m)*	(m AHD)	(m AHD)
20,000	7.04	115.72	114.04
50,200	9.93	118.61	116.93
63,500	10.27	118.95	117.27
100,000	10.84	119.52	117.84
142,000	11.29	119.97	118.29
184,000	11.60	120.28	118.60

** Gauge zero 108.679 m AHD*

3.3.4 Calibration Results

The water levels predicted by the model at the gauge site for each flow are shown in Table 3-6**Error! Reference source not found.**. The level for the 20,000 ML/d flow was 0.16 m lower than the current rating curve, however it was well within the envelope formed by the historic rating curves. The modelled levels were within 0.1 m of the current rating curve for flows from 50,000 to 100,000 ML/d. Above this flow, the modelled levels started to diverge from the rating curve, with the modelled level for a flow of 184,000 ML/d being 0.62 m higher than the rating curve.

Given that gauging only extend up to 100,000 ML/d this is considered to be a good calibration result. The rating curve was well-matched below this flow. This indicates that the extrapolation of the rating curve above this flow may overestimate flows from a given level.

Table 3-6 Calibration results

Flow	Level	Gauge Height	Deviation from current rating curve	Deviation from historic level
(ML/d)	(m AHD)	(m)*	(m)	(m)
20,000	115.56	6.88	-0.16	-
50,200	118.55	9.87	-0.06	-0.27 (2010)
63,500	118.90	10.22	-0.05	-0.05 (1993)
100,000	119.61	10.93	+0.09	-
142,000	120.32	11.64	+0.35	+0.35 (1994)
184,000	120.90	12.22	+0.62	-

** Gauge zero 108.679 m AHD*

3.3.5 Sensitivity Tests

A sensitivity test was undertaken with the tailwater level lowered by 1 m. The tailwater level at the model outlet was set to 2.68 m below the expected Murchison gauge level rather than 1.68 m.

A sensitivity test was undertaken with the roughness values increased by 25% for all land use types.

The sensitivity test results are given in Table 3-7 and are shown compared to the rating curves in Figure 3-6.

The modelled water level at the gauge was found to be moderately sensitive to roughness over the full range of flows, with water levels raised by 0.18 to 0.35 m due to increased roughness.

The sensitivity to tailwater level was significant at low (in-channel) flow, with the level decreased by 0.41 m at a flow of 20,000 ML/d. For flows above 50,000 ML/d, the water level was less sensitive to tailwater levels, with decreases of 0.06 to 0.11 m resulting from the 1 m drop in tailwater level.

Table 3-7 Sensitivity test results

	Increased Roughness		Decreased Tailwater	
Flow (ML/d)	Level (m AHD)	Change (m)	Level (m AHD)	Change (m)
20,000	115.90	+0.34	115.15	-0.41
50,200	118.75	+0.21	118.45	-0.09
63,500	119.08	+0.18	118.84	-0.06
100,000	119.89	+0.28	119.51	-0.10
142,000	120.67	+0.35	120.21	-0.11
184,000	121.21	+0.31	120.81	-0.09

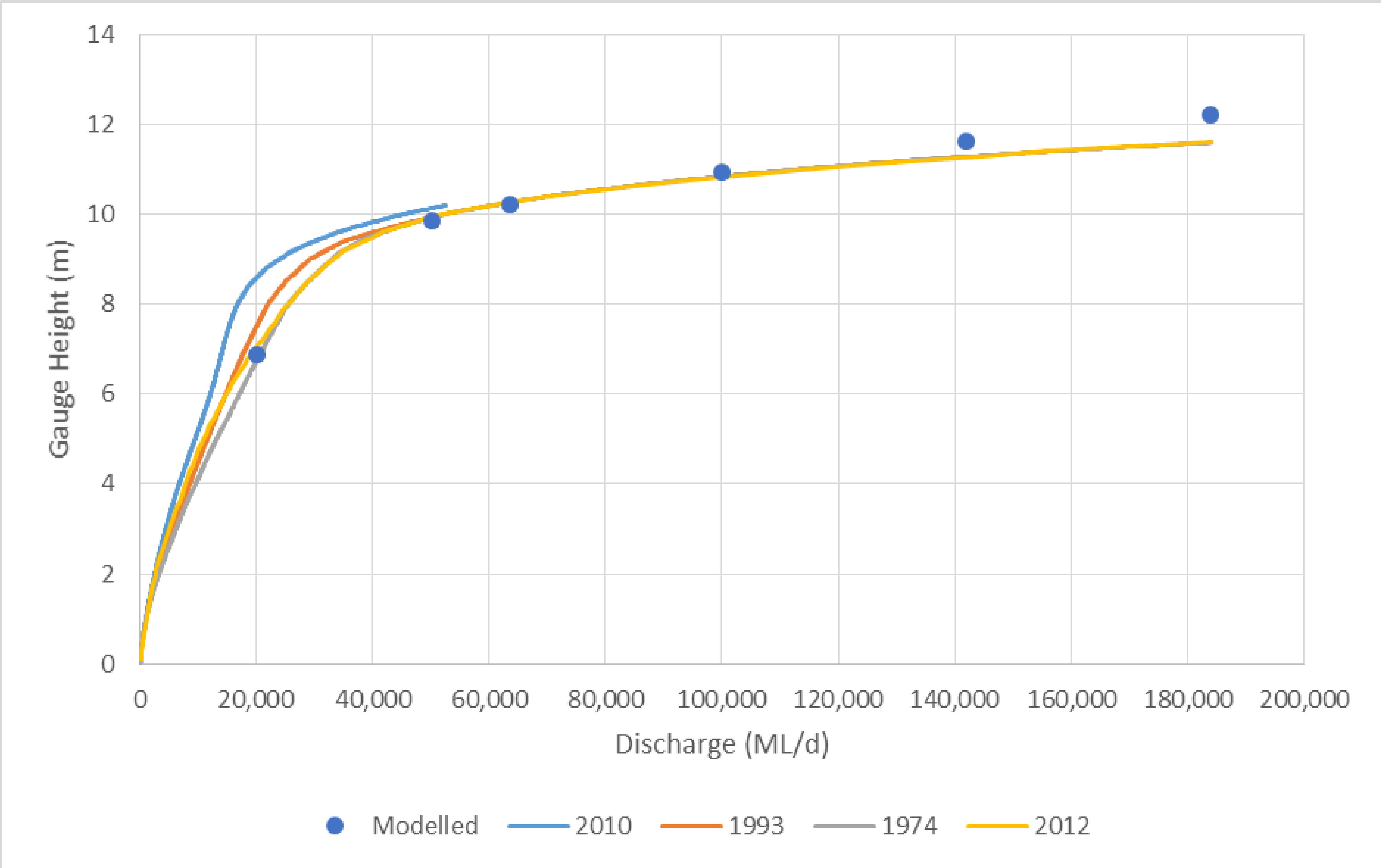

Figure 3-5 **Modelled results versus current and historic rating curves**

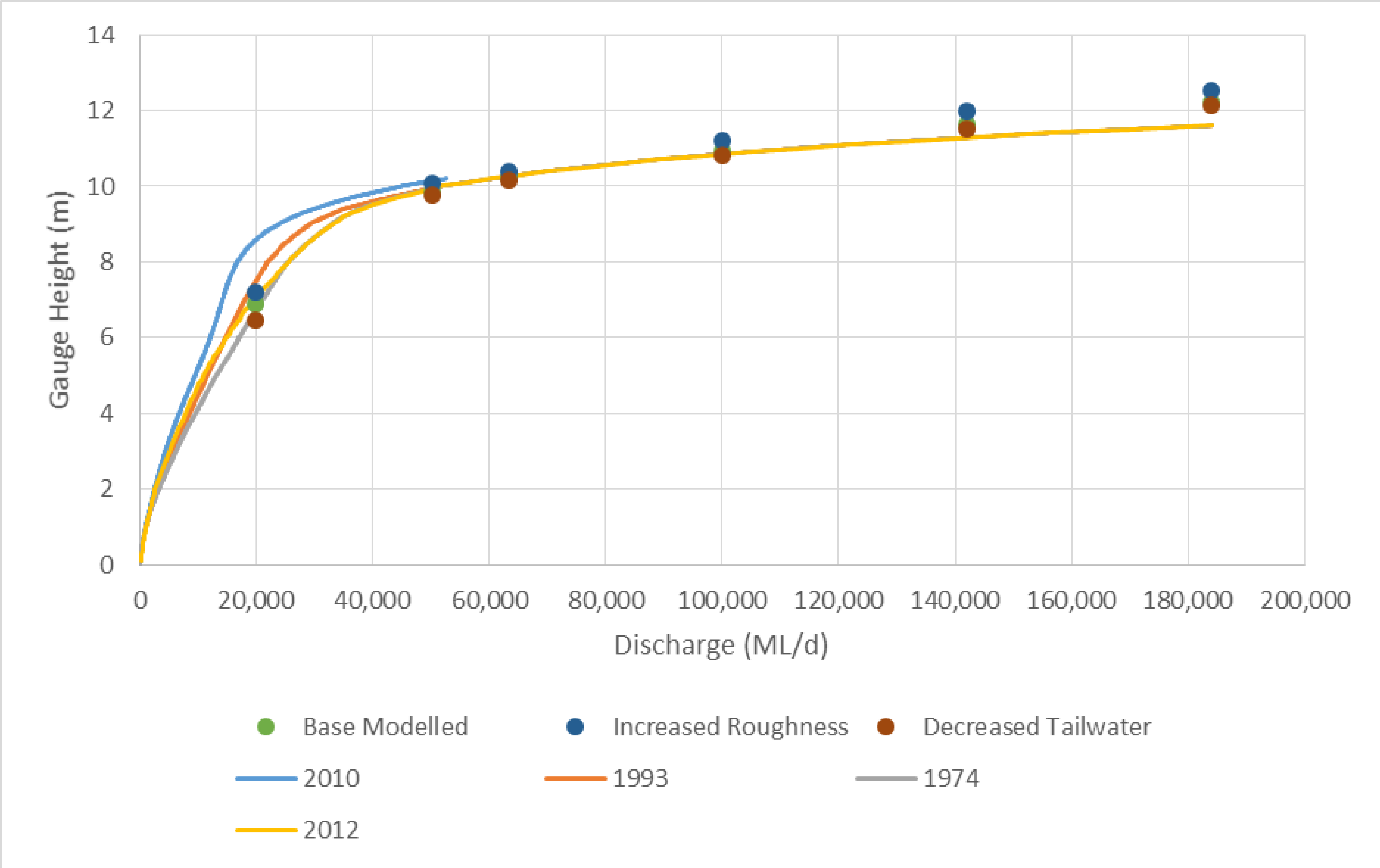

Figure 3-6 **Sensitivity test results versus base modelled results and current and historic rating curves**

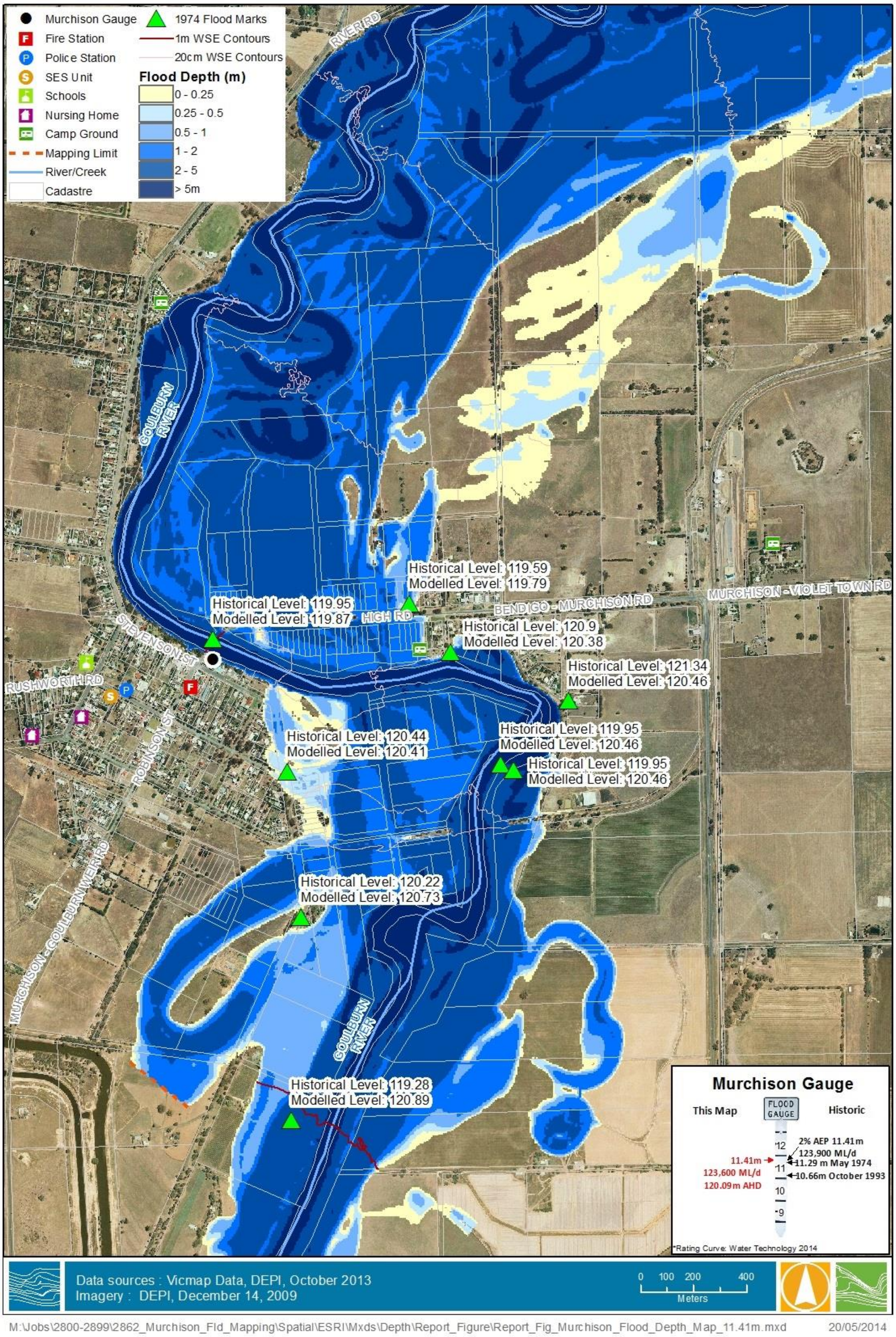

Figure 3-7 Comparison of modelled gauge heights to 1974 historical levels

3.3.6 Comparison to Historical Flooding

The largest historical flood event at Murchison was in 1916, although there is very little information available to compare the modelling results to. The largest recent event was in 1974, reaching a gauge level of 11.29 m, which is close to a 2% AEP flood event. Figure 3-7 shows the modelled flood extent and water level contours for a gauge level of 11.4 m. Historic flood marks are also shown along with estimated modelled flood levels. The 1974 flood event was not modelled as part of this study, so levels were estimated based on the water level results of the 11.2 m and 11.4 m modelled gauge heights. The reliability of the historic flood levels in the Victorian Flood Database was either medium or low, which is evident in the variance of observed levels. Overall the modelled levels fall within the variance of the observed levels.

3.3.7 Calibration Summary

Given the good quality of calibration to the high reliability section of the rating curve and the relative insensitivity to tailwater conditions and roughness above this point, the calibration is considered to be adequate for simulation of design events in the hydraulic model.

4. DESIGN FLOOD MAPPING

4.1 Revised Rating Curve

A revised rating curve was developed using the existing rating curve up to 6 m and the modelled rating curve above 6 m (Figure 4-1).

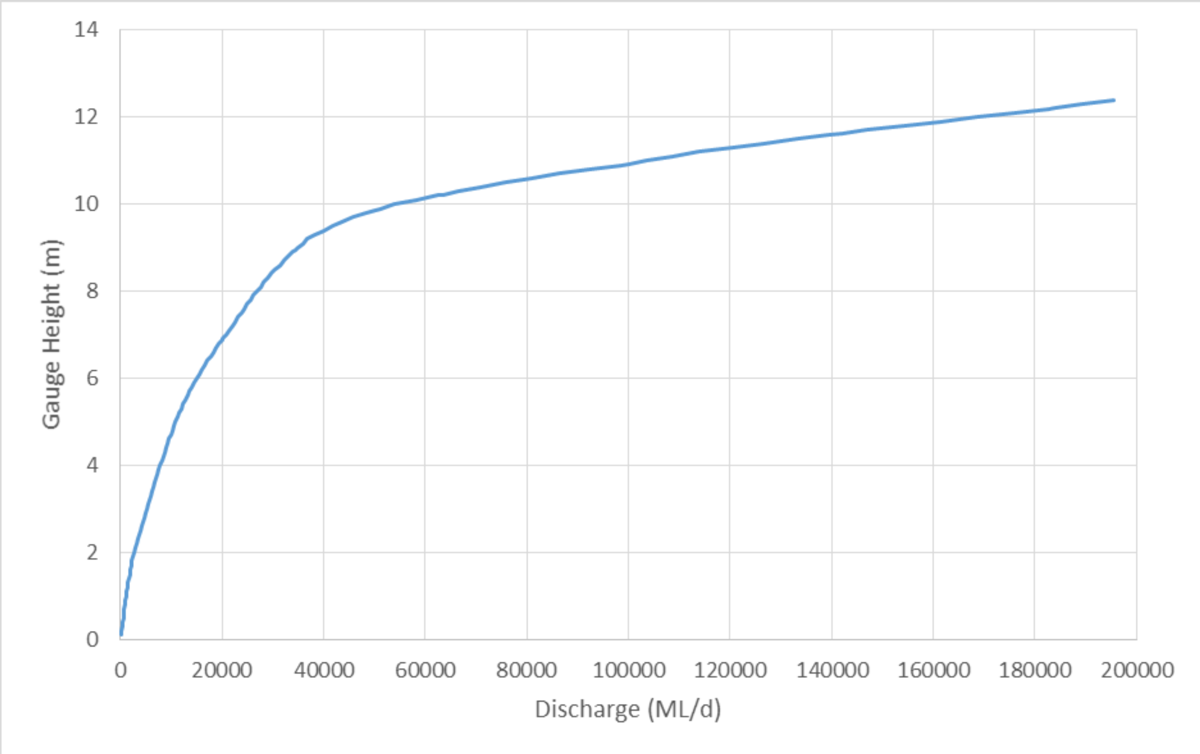

Figure 4-1 Revised rating curve

4.2 Design Flow/Level Scenarios

The design flood mapping is based on 12 incremental gauge heights from 9.0 to 12.2 m. 12 scenarios were specified in the brief, including the 1% AEP gauge height. The revised estimate for the 1% AEP gauge height is 11.9 m, which was derived by applying the updated rating curve from this study. Details of the Flood Frequency Analysis (FFA) undertaken for the Murchison gauge are included in the Shepparton-Mooroopna Flood Mapping and Flood Intelligence Project Hydrology and Hydraulic Calibration Report (Water Technology 2014). The GEV and Log Pearson III distributions were both fitted, with the GEV distribution judged to have a better fit. The results of the revised FFA from that report are shown in Table 4-1, and the depth map produced for the 1% AEP design event is shown in Figure 4-2.

The flows corresponding to each gauge height were derived from the revised rating curve. The 12 flow scenarios are shown in Table 4-2. The tailwater was set to 1.68 m below the gauge height, as per calibration. The depth maps for each corresponding gauge height are shown in Appendix A.

Flood extents for a selection of design events are shown in Figure 4-3, with a description of the flooding consequence for each gauge level summarised in Table 4-3.

Table 4-1 Design peak flows for Goulburn River @ Murchison (405200), revised rating curve data – *source:* Shepparton-Mooroopna Flood Mapping and Flood Intelligence Project *Hydrology and Hydraulic Calibration Report (Water Technology 2014)*

AEP	ARI (1 in X years)	GEV Peak Flow (ML/d) Post-Big Eildon Record 1956-2012 plus 1916 8 low flows censored, 74 flows below 1916 threshold censored	GEV Peak Flow (ML/d) Entire Record 1881-2012 10 low flows censored	Adopted Peak Flow (ML/d)
20%	5	49,100	59,700	49,100
10%	10	69,000	78,600	69,000
5%	20	90,900	97,700	90,900
2%	50	123,900	123,900	123,900
1%	100	152,600	144,700	152,600
0.5%	200	185,200	166,500	166,500
0.2%	500	235,200	196,900	196,900

Table 4-2 Design gauge height scenarios and corresponding flows

Design Gauge Height	Modelled Gauge Height	Flow	Flow	Tailwater
(m)	(m)	(ML/d)	(m^3/s)	(m AHD)
9.0	9.05	34,900	404	116.0
10.2	10.21	62,600	724	117.2
10.7	10.68	86,400	1,000	117.7
10.8	10.80	92,200	1,067	117.8
11.0	11.02	103,500	1,198	118.0
11.2	11.22	114,000	1,319	118.2
11.4	11.41	123,600	1,431	118.4
11.6	11.61	134,700	1,559	118.6
11.8	11.83	147,700	1,709	118.8
11.9 (1% AEP)	11.92	152,600	1,766	188.9
12.0	12.04	160,200	1,854	119.0
12.22 (1916)	12.28	175,300	2,029	119.2

Table 4-3 Cumulative Flooding Consequences for Design Gauge Heights

Design Gauge Height (m)	Flow (ML/d)	Annual Exceedance Probability	Cumulative Consequence
9.0	34,900	50% AEP (<2 year ARI)	Low lying rural properties upstream and downstream of Murchison are likely to be flooded.
10.2	62,600	12% AEP (8 year ARI)	Overland flooding south of High Road covering western side of River Haven caravan park. Extensive inundation of floodplain and shallow water over Willoughby Street south of Station Street.
10.7	86,400	6% AEP (18 year ARI)	Shallow inundation of Hutchinson Road, Old Weir Road and more extensive inundation of Willoughby Street south of Station Street.
10.8	92,200	5% AEP (20 year ARI)	Inundation of properties on east side of Willoughby Street, including cemetery, south of Station Street.
11.0	103,500	3.3% AEP (30 year ARI)	Inundation of several properties on east of Willoughby Street near Watson Street.
11.2	114,000	2.5 % AEP (40 year ARI)	Inundation of Willoughby Street between Watson Street and Stevenson Street and adjacent properties. Inundation of Watson Street east of Willoughby Street.
11.4	123,600	2% AEP (50 year ARI)	Further inundation of properties east of Willoughby Street between Watson Street and Station Street. Flow across Donegans Road north of Hutchinson Road.
11.6	134,700	1.4% AEP (70 year ARI)	Flow across Watson Street west of Willoughby Street.
11.8	147,700	1.1% AEP (90 year ARI)	Inundation of additional property west of Willoughby Street between Watson Street and Stevenson Street. Extensive flow across Donegans Road.
11.9	152,600	1% AEP (100 year ARI)	Flow will begin to overtop Murchison-Bendigo Road causeway.
12.0	160,200	0.7% AEP (150 year ARI)	Breakout flow across Gillam Road towards Hutchinson Road across several properties.
12.22 (1916)	175,300	0.3% AEP (300 year ARI)	Breakout flow through town flowing across Robinson Street south of Station Street, then across Station Street, Watson Street and Stevenson Street between Impey Street and Robinson Street. Extensive flooding of River Haven Caravan Park.

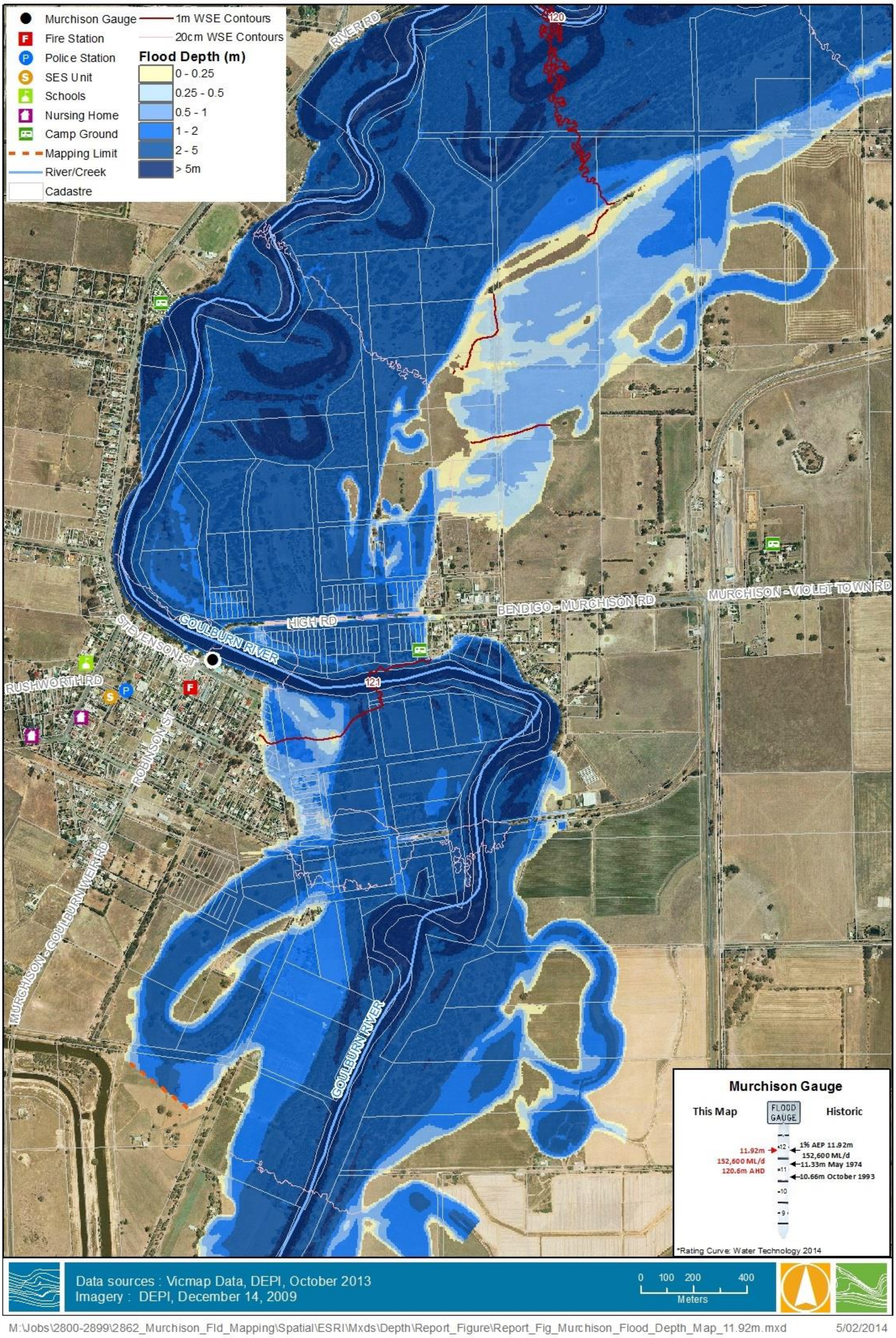

Figure 4-2 1% AEP Design Flood Event Depth Map for Murchison

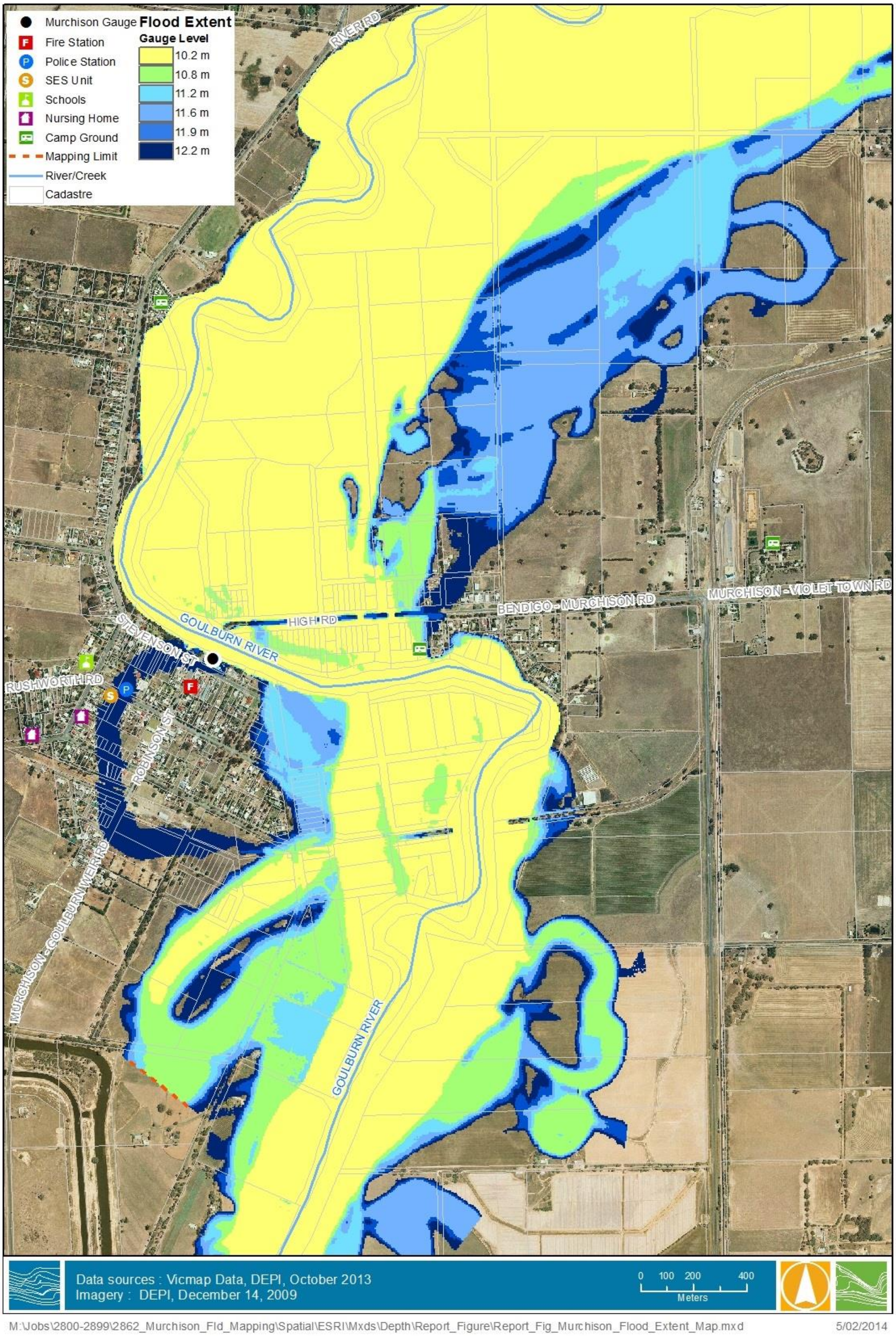

Figure 4-3 Flood Extents for Selected Design Flood Events

4.3 Discussion

The design flooding scenarios for the range of gauge heights has produced water levels for the Goulburn River around Murchison that can be used to assess any flood related impacts and any possible mitigation measures.

Levee Freeboard

There's a small levee just downstream of the Bendigo-Murchison Road bridge on the left bank of the Goulburn River, this is shown in Figure 4-4. This levee provides protection from river flooding, particularly against back flooding along the depression that runs past Stevenson Street back towards Watson Street. This levee has a minimum crest elevation of 121.04 m AHD. The largest simulated flood event was the 1916 flood level (gauge level of 12.22 m), which corresponds to 120.90 m AHD upstream of the bridge where the gauge is located. Downstream of the bridge near the levee, the water level in the river for this scenario was 120.73 m AHD, which means the levee has a freeboard of 310 mm above the highest recorded flood level. The 1% AEP has a water level near the levee of 120.43 m AHD, which means the levee has a freeboard of 610 mm about the 1% AEP event.

Recommendations:

- This levee provides adequate protection for the town, no upgrades are required.

Figure 4-4 View of levee in Murchison Township looking east along Stevenson Street

Surcharge through Town Drainage

The drainage pipes in Murchison that surcharge to the Goulburn River are assumed not to have any flood valve's installed. There is a possibility that surcharge from the river may inundate low lying areas of the town during large flood events. Figure 4-5 shows the areas of the town for the five largest modelled flood scenarios that are below the water level of the river at the pipe outfall locations. Only the areas that are connected to a drainage pit have been mapped. It is likely that there will be some head loss in the pipes as the water surcharges, therefore this map is a conservative estimate of possible risk to flooding in the town via the stormwater system backing up from the river.

Recommendations:

- Inspect town drainage pipes at Goulburn River outfall to determine presence of any flood valves; and
- If no flood valves are present, consider the installation of either flood valves or vertical gates/penstocks.

Flood Mitigation

For the largest modelled event, the hydraulic model shows a flow path through the western side of Murchison. There is a potential for inundation of several properties through the town during this very large event. The source of this flow path is due to a breakout from the Goulburn River upstream of town through a shallow depression. Given the long lead times prior to a Goulburn River flood in Murchison, there is time to coordinate sandbagging of this breakout to prevent flooding through town. Figure 4-6 shows the recommended sandbagging location at the upstream end of the flow path. This location appears to be on private land, therefore an arrangement with the landholder will need to be agreed ahead of time. It is also suggested that this mitigation scenario be modelled and if found to have no adverse impact on surrounding properties then a permanent low level inexpensive earthen bund could be considered to permanently block this flow path.

Recommendations:

- Investigate the impact of a permanent earthen bund.

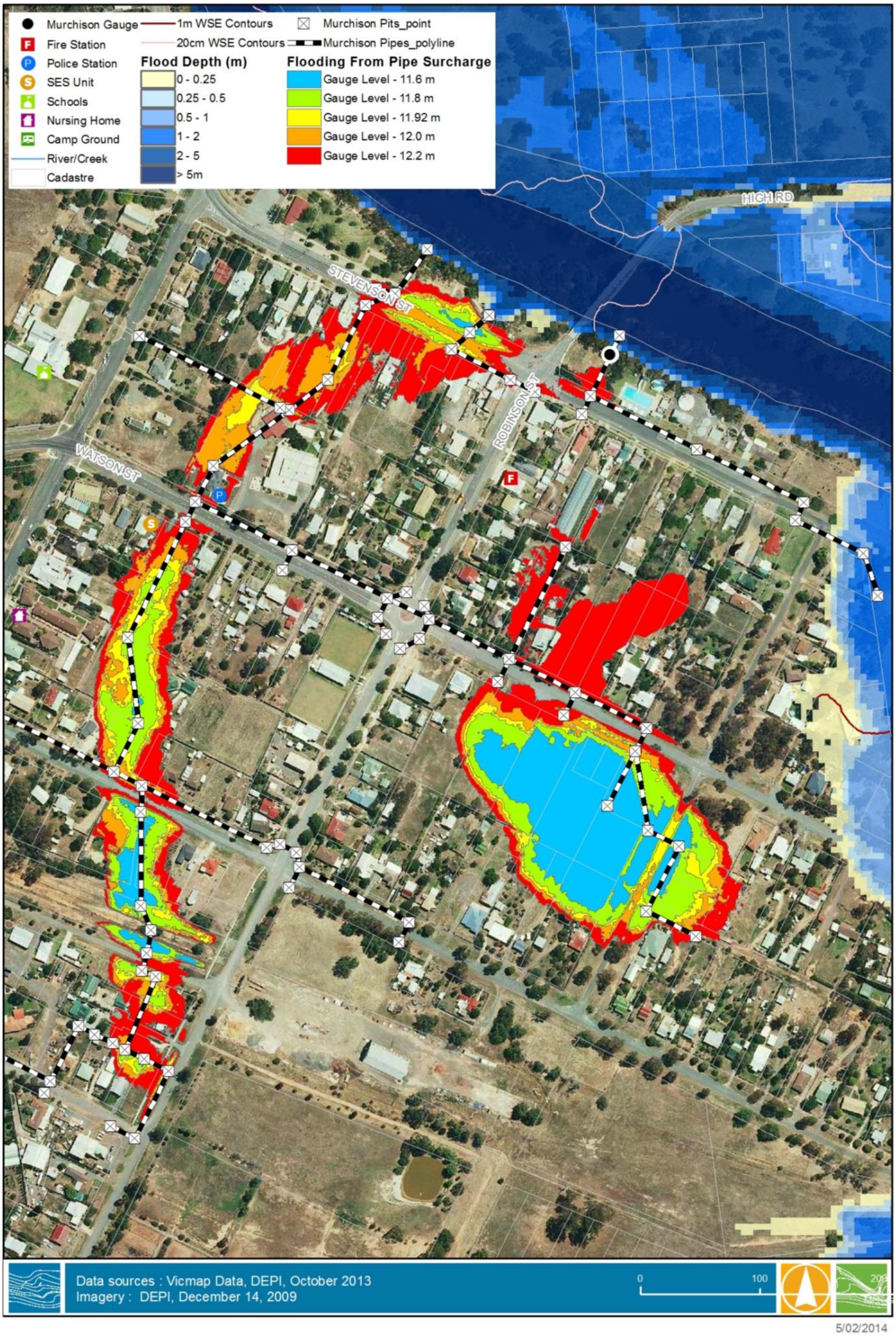

Figure 4-5 Possible Areas of Flooding Due to Pipe Surcharge

Figure 4-6 Recommended Sandbagging Location for Extreme Flood Events

APPENDIX A DEPTH MAPS

APPENDIX B STRUCTURE DETAILS